BARACKMAN

Investigation of a Highway Bridge

Civil Engineering
B. S.

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INVESTIGATION OF A HIGHWAY BRIDGE

BY

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THESIS FOR DEGREE OF BACHELOR OF SCIENCE
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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

Guy Bernard Barackman

ENTITLED Investigation of a Highway Bridge

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE

OF Bachelor of Science in Civil Engineering

Ira O. Baker

HEAD OF DEPARTMENT OF Civil Engineering.

Investigation of a Highway Bridge.

This bridge is located at Mahomet, Illinois and consists of one 100-foot and two 75-foot arch spans. In this investigation the 100-foot span only will be considered since the efficiency of the bridge probably depends on the efficiency of the longest span.

Plates I, II, and III show three photographic views of the bridge. The makeup of the several members of the main trusses are shown in the diagram on page 2.

PLATE I.

PLATE II.

PLATE III.

Composition of Members of Main Trusses

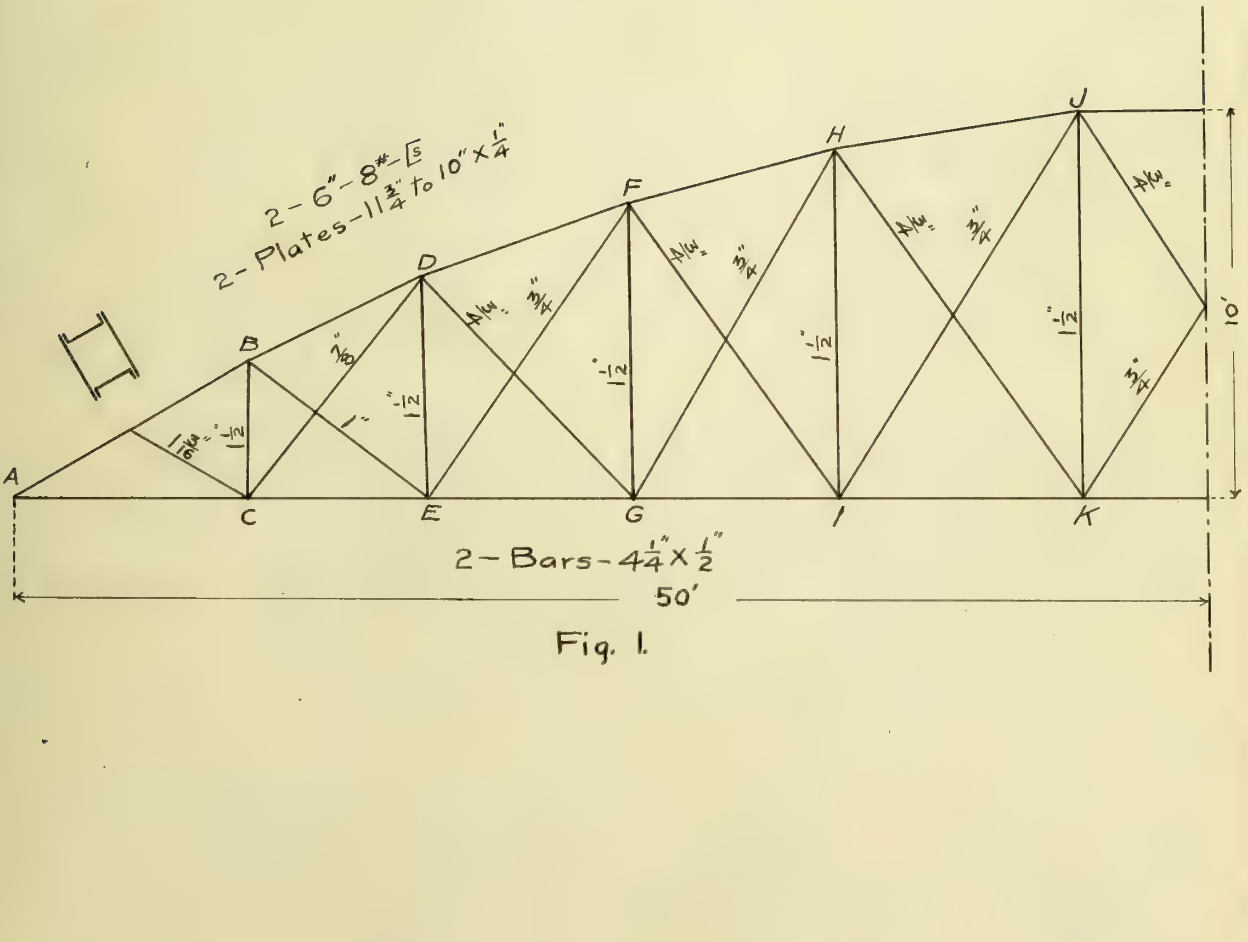
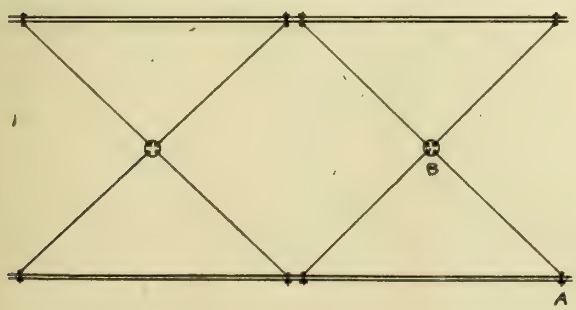


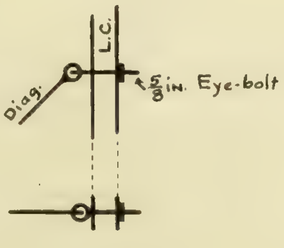
Fig. 1.

Bottom Lateral System.

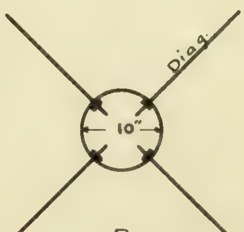
In the bottom lateral system the floor beams act as struts and the diagonal members are round rods. The rods connect with the lower chord by means of eye-bolts which pass through both bars and have nuts on the outside. This connection is shown at A below. Where the lateral rods intersect there is a ten inch ring $\frac{3}{8}$ inches thick and $1\frac{1}{2}$ inches wide. The rods pass inside of this ring and are held there by a nut. This connection is shown at B below.

The floor beams being notched down on the lower chord, give considerable lateral stiffness so that with the help of the diagonals the system is sufficient.





A



B

Computation of Dead Load.

The dead load was computed from actual measurements of the bridge. In the computation of the stresses the dead load was considered as being uniformly distributed. The details of these computations are as follows:

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length Ft.	Weight in lbs. per Ft.	Weight	Total Weight
1	Arch - 2 Pieces						
	[	2	6"	35.85	8.0	574	
	[	2	6"	37.13	8.0	594	
	[	2	6"	30.25	8.0	484	
	[	2	6"	35.25	8.0	564	
	[	2	6"	36.63	8.0	586	
	[	2	6"	29.75	8.0	476	
	Plates	16	$4\frac{3}{4}" \times \frac{1}{4}"$	.79	4.04	51	
	do.	8	$11\frac{3}{4}" \times 10" \times \frac{1}{4}"$	51.61	9.25	3819	
	Rivet Heads	4532	$\frac{1}{2}"$		.048	218	
	[	4	6"	23.0	8.0	736	8102

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length Ft.	Weight in lbs. per ft.	Weight	Total Weight
2	Lower Chord – 2 Pieces						
	Flats	4	$4\frac{1}{4}'' \times \frac{1}{2}''$	95.0	7.22	2744	
	Splice Plates	24	$4\frac{1}{4}'' \times \frac{1}{2}''$	0.88	7.22	152	
	End Rods	4	$1\frac{1}{2}''$	3.0	6.01	72	
	Castings	20			10.0	200	
	Shoes	4			90.0	360	
	Rivet Heads	144	$\frac{5}{8}''$		0.1	14	3542
3	Diagonals – 40 Pieces						
	Rods	4	$1\frac{3}{16}''$	5.08	3.77	77	
	"	4	$1''$	7.71	2.67	82	
	"	4	$\frac{7}{8}''$	8.67	2.04	71	
	"	4	$\frac{3}{4}''$	9.92	1.50	60	
	"	4	$\frac{3}{4}''$	11.08	1.5	67	
	"	4	$\frac{3}{4}''$	12.63	1.5	76	
	"	4	$\frac{3}{4}''$	11.54	1.5	69	
	"	4	$\frac{3}{4}$	13.38	1.5	80	
	"	4	$\frac{3}{4}''$	14.13	1.5	87	
	"	4	$\frac{3}{4}''$	14.75	1.5	89	758
4	Verticals – 20 Pieces						
	Rods	4	$1\frac{1}{2}''$	4.83	6.01	116	
	"	4	$1\frac{1}{2}''$	6.96	6.01	168	
	"	4	$1\frac{1}{2}''$	9.00	6.01	216	
	"	4	$1\frac{1}{2}''$	9.92	6.01	238	
	"	4	$1\frac{1}{2}''$	10.55	6.01	253	991

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length Ft.	Weight in lbs. per ft.	Weight	Total Weight
5	Sway Braces – 6 Pieces.						
	Rods	4	$1\frac{1}{2}''$	8.0	6.01	192	
	"	2	$1\frac{1}{2}''$	10.50	6.01	126	
	Connections	6	$\frac{3}{8}'' \times 1\frac{3}{4}''$	0.50	2.23	7	
	Bolts	6	$\frac{5}{8}''$	0.25	0.54	3	
	⊏	3	$6''$	20.33	8.00	488	816
6	Bottom Lateral						
	Rods	24	$\frac{5}{8}''$	20.3	1.04	507	
	Rings	6	$1\frac{1}{2}'' \times \frac{3}{8}''$	2.62	1.92	30	537

Total weight of metal = 14746

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length Ft.	Board Feet	Wt. per B.M.	Weight	
7	Stringers	56	$3''\times4''$	14	784	4.16	3261	
8	Joists	50	$2''\times12''$	15	1500	4.16	6240	
9	Flooring	100	$2''\times12''$	15	3000	4.16	12480	
10	Hub Guards	6	$2''\times8''$	16	128	2.00	256	
11	Felloe "	3	$3''\times4''$	16	48	4.16	200	
12	Sway Bracing	3	$2''\times6''$	20.33	61	4.16	254	22691

Total weight of lumber = 22691

" " " metal = 14746

" " " bridge = 37437

Total weight per lineal foot of bridge = 374 lbs.

" " " " " " truss = 187 "

Live Load

The live load is assumed as seventy pounds per square foot of floor surface. The computations of live loads for each panel are as follows:

Total live load = 100 × 14 × 70 = 98000 lbs.

" " " per foot of bridge = 980 lbs.

" " " " " of truss = 490 "

PANEL LOADS AND REACTIONS.

Panel	Panel Length	Whole Panel Load	Half Panel Load	Joint	Joint Load	Successive Sums	Reaction at Remote Abutment
A–C	10′–1½″	4960	2480				
C–E	7–0¼	3440	1720	C	4200		425
E–G	8–4¼	4100	2050	E	3770	7970	1320
G–I	8–11	4370	2180	G	4230	12200	2170
I–K	10–1	4940	2470	I	4650	16850	3740
K–K′	11–1¼	5440	2720	K	5190	22040	6000
K′–I′	10–1	4940	2470	K′	5190	27230	8930
I′–G′	8–11	4370	2180	I′	4650	31880	12060
G′–E′	8–4¼	4100	2050	G′	4230	36110	15290
E′–C′	7–0¼	3440	1720	E′	3770	39880	18560
C′–A′	10–1½	4960	2480	C′	4200	44080	22020

Stresses.

Maximum stresses in the several members.

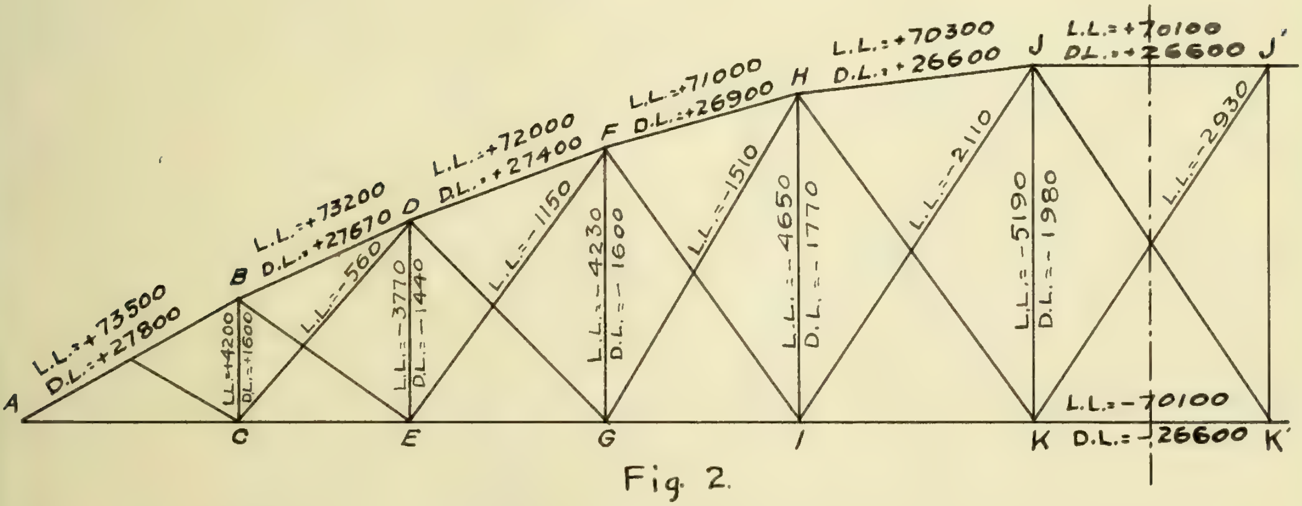


Fig. 2.

Live load = 70 lbs. per square foot of floor.

Live load = 980 lbs. per lineal foot of bridge.

" " = 490 " " " " " truss.

Dead " = 374 " " " " " bridge.

" " = 187 " " " " " truss.

Efficiency of Members.

The term efficiency will be used to indicate the ratio of the allowable stress to the actual stress. Of course for a well designed bridge the efficiency of all members should be one or a little more than one.

The efficiency will be computed according to Cooper's Specifications for Highway Bridges, 1901 edition. This investigation will be made for the following conditions:

1.- Dead load and a live load of seventy pounds per square foot of clear roadway.
2.- Dead load and a traction engine weighing 8000 pounds.
3.- Dead load and two farm wagons loaded with 6000 pounds each.
4.- Dead load and one such farm wagon.

Investigation of Arch.

Preliminary to computing the efficiencies of the arch it is necessary to compute the area, moment of inertia, etc. of the several sections. These are shown in Tables 1 and 2 following.

The arched upper chord has a depth of $11\frac{3}{4}$ inches at its ends and of 10 inches at its middle. In investigating a section of the upper chord, average values of the moments of inertia, areas, etc., at the ends of the section were used. Figure 3 is given to make clear the nomenclature.

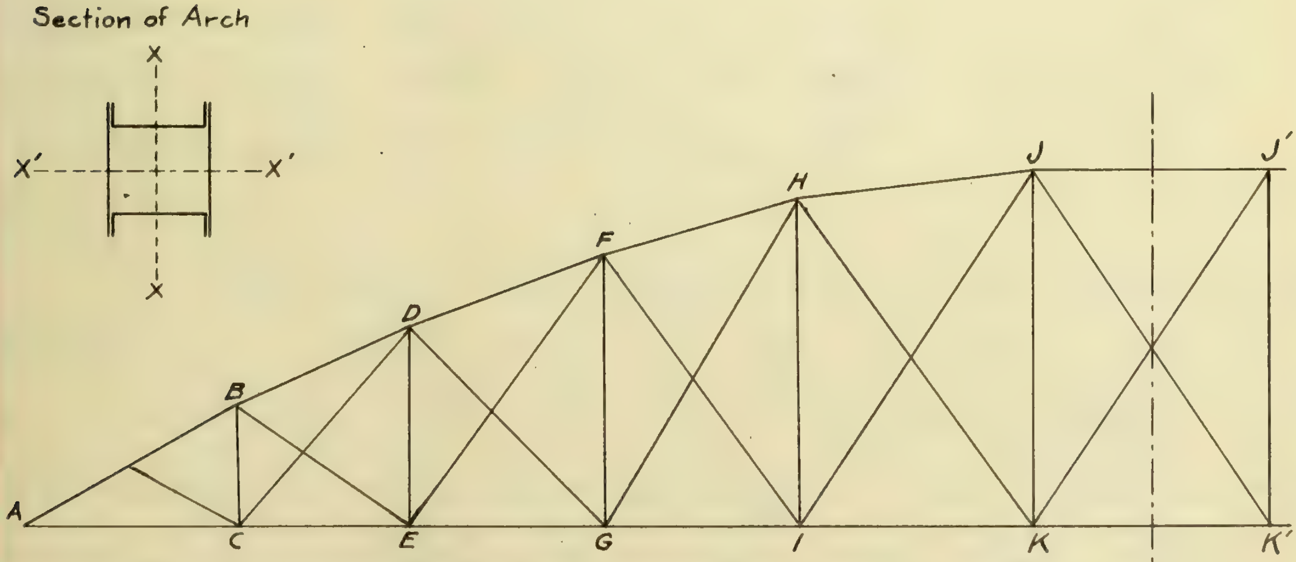


Fig. 3. Nomenclature to Designate Parts of the Bridge.

Table 1.

Dimensions of the Arch at the Several Joints.

Joint	Area Sq. In.	I Axis x–x	I′ Axis X′–X′	Rad. of Gyr. r
A	10.64	83.45	164.34	2.80
B	10.52	82.28	154.82	2.80
D	10.26	78.87	136.88	2.77
F	10.14	78.57	128.37	2.78
H	10.08	77.98	124.26	2.78
J	9.82	75.44	112.31	2.77
J′	9.82	75.44	112.31	2.77

Table 2.

Mean Dimensions for the Several Sections of the Arch.

Section	Eccentricity	Area Sq. In.	I Axis X–X	I′ Axis X′–X′	Rad. of Gyr. r	Length Inches	$\frac{l}{r}$
A–B	0.0	10.58	82.86	159.58	2.80	126	45.00
B–D	0.5	10.39	80.57	145.85	2.78	88	31.66
D–F	0.5	10.20	78.72	132.62	2.77	103	37.18
F–H	0.75	10.11	78.27	126.31	2.77	108	39.00
H–J	1.00	9.95	76.71	118.28	2.77	121	43.68
J–J′	2.00	9.76	74.86	108.53	2.77	133	48.02

Since the upper chord is practically continuous from end to end, it is unnecessary to investigate it for the last three conditions of loading mentioned on page   i. e., it is necessary to investigate it for only live load.

Section AB.

A = 10.58 sq. in. I = 82.86 r = 2.80 l = 126 $\frac{l}{r}$ = 45.0 e = 0.0

Allowable L.L. stress = 12000 − 55 × 45 = 9500 lbs. per sq. in.

" D.L. " = 24000 − 110 × 45 = 19050 " " " "

Actual L.L. stress = 73,500 lbs.

" D.L. " = 27800 " .

Required L.L. area = $\frac{73500}{9500}$ = 7.74 sq. in.

" D.L. " = $\frac{27800}{19050}$ = 1.46 " "

Total required area = 9.20 " "

$\frac{10.58}{9.20}$ = 1.15 Efficiency.

Section BD.

A = 10.39 sq. in. I = 80.57 r = 2.78 l = 88 $\frac{l}{r}$ = 31.66 e = 0.5

Allowable L.L. stress = 12000 − 55 × 31.66 = 10260 lbs. per sq. in.

" D.L. " = 24000 − 110 × 31.66 = 20520 " " " "

Actual L.L. = 73200 lbs. Required L.L. area = $\frac{73200}{10260}$ = 7.13 sq. in.

" D.L. = 27670 " " D.L. " = $\frac{27670}{20520}$ = 1.35 " "

Total required area = 8.48 " "

Stress due to eccentricity.

$$f_1 = \frac{100870 \times 0.5 \times 5.625}{145.85 - \frac{100870 \times (88)^2}{32 \times 28000000}} = 2170 \text{ lbs. per sq. in.}$$

$\frac{13200}{11870}$ = 1.11 Efficiency.

Total allowable stress = $\frac{100870}{8.48}$ × 1.1 = 13200 lbs. per sq. in.

10 per cent of total allowable stress = 1320.0 "

Since 2170 > 1320, this section does not meet Cooper's specifications.

Section DF.

$A = 10.20$ sq. in. $I = 78.72$ $r = 2.77$ $l = 103$ in. $\frac{l}{r} = 37.18$ $e = 0.5$ in.

Allowable L.L. stress $= 12000 - 55 \times 37.18 = 9960$ lbs. per. sq. in.

" D.L. " $= 24000 - 110 \times 37.18 = 19920$ " " " "

Actual L.L. stress $= 72000$ lbs. Required L.L. area $= \frac{72000}{9960} = 7.23$ sq. in.

" D.L. " $= 27400$ " " D.L. " $= \frac{27400}{19920} = \underline{1.38}$ " "

Total required area $= 8.61$ " "

Stress due to eccentricity.

$y' = 5.44$ in. $I' = 132.62$.

$M = 99400 \times 0.5 = 49700$ inch lbs.

$$f_1 = \frac{49700 \times 5.44}{132.62 - \frac{99400 \times (103)^2}{32 \times 28\,000\,000}} = 2200 \text{ lbs. per sq. in.}$$

Total allowable stress $= \frac{99400}{8.61} \times 1.1 = 12700$ lbs. per sq. in.

10 percent of total allowable stress $= 1270$ "

Total actual stress $= \frac{99400}{10.20} + 2200 = 12000$ " " " "

$\frac{12700}{12000} = 1.06$ Efficiency.

Since $2200 > 1270$, this section does not meet Cooper's specifications.

Section FH.

$A = 10.11$ sq. in. $I = 78.27$ $r = 2.77$ $l = 108$ in. $\frac{l}{r} = 39.0$ $e = 0.75$ in.

Allowable L.L. stress $= 12000 - 55 \times 39.0 = 9850$ lbs. per sq. in.

" D.L. " $= 24000 - 110 \times 39.0 = 19700$ " " " "

Actual L.L. stress $= 71000$ lb. Required L.L. area $= \frac{71000}{9850} = 7.21$ sq. in.

" D.L. " $= 26900$ " " D.L. " $= \frac{26900}{19700} = 1.37$ " "

Total required area $= 8.58$

Stress due to eccentricity.

$y' = 5\frac{3}{8}''$ $I' = 126.31$

$M = 97900 \times 0.75 = 73500$ inch lbs.

$$f_1 = \frac{73500 \times 5.375}{126.31 - \frac{97900 \times (108)^2}{32 \times 28\,000\,000}} = 3380 \text{ lbs. per sq. in.}$$

Total allowable stress $= \frac{97900}{8.58} \times 1.1 = 12400$ lbs. per sq. in.

10 per cent of total allowable stress $= 1240$ lbs.

Total actual stress $= \frac{97900}{10.11} + 3380 = 12960$ lbs. " " "

$\frac{12400}{12960} = .96$ Efficiency.

Since $3380 > 1240$, this section does not meet Cooper's specifications.

Section HJ.

$A = 9.95$ sq. in. $I = 76.71$ $r = 2.77$ $l = 121$ in. $\frac{l}{r} = 43.68$ $e = 1.0$ in.

Allowable L.L. stress $= 12000 - 55 \times 43.68 = 9600$ lbs. per sq. in.

" D.L. " $= 24000 - 110 \times 43.68 = 19200$ " " " "

Actual L.L. stress $= 70300$ lb. Required L.L. area $= \frac{70300}{9600} = 7.32$ sq. in.

" D.L. " $= 26600$ " " D.L. " $= \frac{26600}{19200} = \underline{1.38}$ " "

Total required area $= 8.70$ " "

Stress due to eccentricity.

$y' = 5\frac{3}{8}$ in. $I' = 118.28$

$M = 96900 \times 1.0 = 96900$ inch lbs.

$$f_1 = \frac{96900 \times 5.375}{118.28 - \frac{96900 \times (121)^2}{32 \times 28\,000\,000}} = 5000 \text{ lbs. per sq. in.}$$

Total allowable stress $= \frac{96900}{8.7} \times 1.1 = 12200$ lbs. per sq. in.

10 per cent of total allowable stress $= 1220$ "

Total actual stress $= \frac{96900}{9.95} + 5000 = 14700$ " " " "

$\frac{12200}{14700} = .83$ Efficiency.

Since $5000 > 1220$, this section does not meet Cooper's specifications.

Section JJ'.

$A = 9.76$ sq. in. $I = 74.86$ $r = 2.77$ $l = 133$ in. $\frac{l}{r} = 48.02$ $e = 2.0$ in.

Allowable L. L. stress $= 12000 - 55 \times 48.02 = 9360$ lbs. per sq. in.

" D. L. " $= 24000 - 110 \times 48.02 = 18720$ " " " "

Actual L. L. stress $= 70100$ lbs. Required L. L. area $= \frac{70100}{9360} = 7.49$ sq. in.

" D. L. " $= 26600$ " " D. L. " $= \frac{26600}{18720} = 1.42$ " "

Total required area $= 8.91$ " "

Stress due to eccentricity.

$y' = 5\frac{5}{16}$ in. $I' = 108.53$

$M = 96700 \times 2.0 = 193400$ inch lbs.

$$f_1 = \frac{193400 \times 5.3125}{108.53 - \frac{96700 \times (133)^2}{32 \times 28\,000\,000}} = 10300 \text{ lbs. per sq. in.}$$

Total allowable stress $= \frac{96700}{8.91} \times 1.1 = 11900$ lbs. per sq. in.

10 per cent of total allowable stress $= 1190$ " "

Total actual stress $= \frac{96700}{9.76} + 10300 = 20200$ " " " "

$\frac{11900}{20200} = .59$ Efficiency.

Since $10300 > 1190$, this section does not meet Cooper's specifications.

Investigation of Verticals.

The vertical members were investigated for cases 1 and 2.

Vertical members are all rods $1\frac{1}{2}$ inches in diameter, area 1.767 sq. in.

Case 1.

JK. Maximum uniform load occurs at center of bridge where panel length is greatest.

L.L. = 490 x 10.6 = 5194 lb. Allowed L.L. stress = 12500 lb.

D.L. = 138 x 10.6 = 1980 "   "   D.L. " = 25000 "

Required L.L. area = $\frac{5194}{12500}$ = 0.42 sq. in.

"   D.L. " = $\frac{1980}{25000}$ = .08 " "

Total required area = 0.50 " "

$\frac{1.767}{0.50}$ = 3.53 Efficiency

Case 2.

8000 lb. Traction engine. Weight on rear wheel = 5560

Maximum floor beam reaction = 4370 lb.

Required L.L. area = $\frac{4370}{12500}$ = 0.35 sq. in.

"   D.L. " = $\frac{1980}{25000}$ = .08 " "

Total required area = 0.43 " "

$\frac{1.767}{0.43}$ = 4.1 Efficiency.

Investigation of Lower Chord.

The lower chord was investigated for all four cases of loading.

Case 1.

Section JJ'. 2-Bars-$4\frac{1}{4} \times \frac{1}{2}$

$I = 6.4$ $A = 4.25$ sq. in. Net area $= A' = 3.63$ sq. in.

$\frac{A}{A'} = \frac{4.25}{3.63} = 1.17$

Uniform L.L. = 70100 lbs.

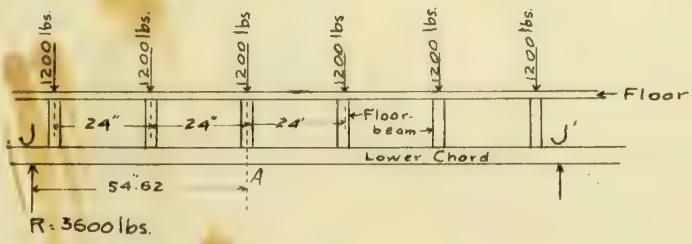
D.L. = <u>26600</u> "

Total direct stress = 96700 "

Unit " " $= f_2 = \frac{96700}{3.63} = 26600$ lbs. per sq. in.

Cross bending stress.

Since the floor beams rest directly on the lower chord there will be stress in the latter due to cross bending.



Live load = 490 lbs. per lineal foot of truss = 41 lbs. per lin. inch of truss.

Dead " = weight of flooring, stringers. <u>9</u> " " " " "

Total load per lineal inch of truss = 50 lbs.

Load on one floor beam = $24 \times 50 = 1200$ lbs.

Reaction at J = R = 3600 lbs.

Moment at A = $3600 \times 54.62 - 2400 \times 36 = 110230$ inch lbs.

$$f_1 = \frac{110230 \times 2\frac{1}{8}}{6.4 + \frac{26600 \times (133.3)^2}{280\,000\,000}} = 28990 \text{ lbs. per sq. in.}$$

Total actual stress $= f_1 + f_2$

$= 28990 + 26600$

$= 55590$ lbs. per sq. in.

Required L.L. area $= \frac{70100}{12500} = 5.68$ sq. in.

" D.L. " $= \frac{26600}{25000} = \underline{1.06}$ " "

Total required area $= 6.74$ " "

Where, allowable L.L. stress $= 12500$ lbs. per sq. in.

and " D.L. " $= 25000$ " " " "

Total allowable stress $= \frac{96700}{6.74} = 14350$ " " " "

$\frac{14350}{55590} = .25$ Efficiency.

Dead load only.

Load on each floor beam $= 9 \times 24 = 216$ lbs.

Reaction at J = R = 648 lbs.

Moment at A $= 648 \times 54.62 - 432 \times 36 = 19850$ inch lbs.

$$f_1 = \frac{19850 \times 2\frac{1}{8}}{6.4 + \frac{26600 \times (133.3)^2}{280\,000\,000}} = 5220 \text{ lbs. per sq. in.}$$

$$f_2 = \frac{26600}{3.63} = 7300 \text{ lbs. per sq. in.}$$

Total dead load stress $= f_1 + f_2$

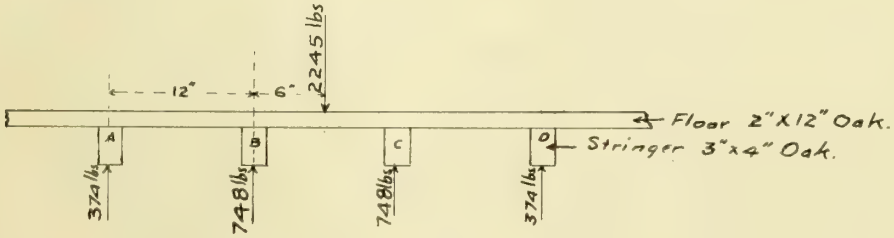
$= 5220 + 7300$

$= 12520$ lbs. per sq. in.

Case 2.

If an 8000-lb. traction engine crosses the bridge, the maximum floor beam reaction on the side to which the engine is closest will be about, $\frac{2}{3} \times 8000 \times \frac{11}{14} = 4490$ lbs. This is considering that the engine is as close to the side of the bridge as it can get. The load will will be transferred by the flooring to the stringers as follows:

Weight on both rear wheels = 4490 lbs.
" " one " " = 2245 "



Stringers A and D each take $\frac{1}{6}$ of the load.
" B " C " " $\frac{1}{3}$ " " "

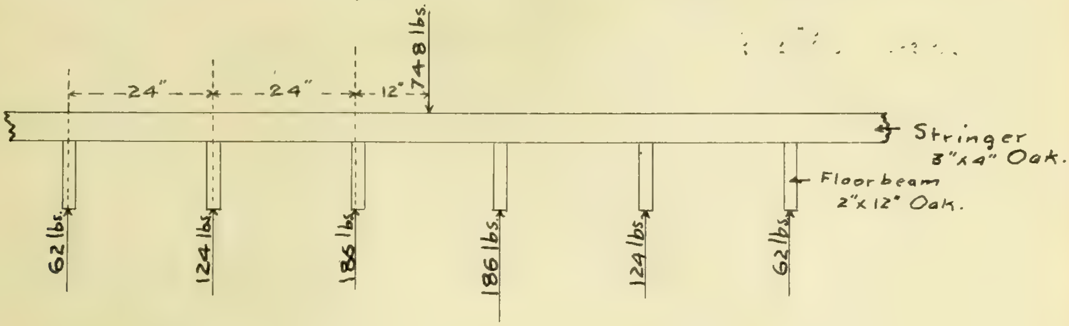
Maximum bending moment in the flooring is under the load.

$M = 748 \times 6 + 374 \times 18 = 11110$ inch lbs.

$S = \frac{11110 \times 1}{8} = 1390$ lbs. per sq. in.

This is safe since with a factor of safety of ten the allowable stress on white oak in tension is 1000 lbs. per sq. in.

From the stringers the load will be transferred to the floor beams thus:

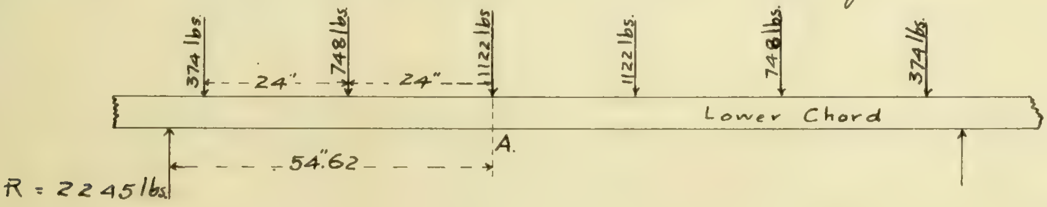


Moment under the load:

$186 \times 12 + 124 \times 36 + 62 \times 60 = 10410$ inch lb.

$S = \frac{10410 \times 2}{16} = 1300$ lbs. per sq. in.

The flooring, stringers and floor beams distribute the load so that it is applied to the lower chord at six points, with the heaviest portions of the load near the middle. The loading on the lower chord is shown in the diagram.



Maximum moment in lower chord is at A, at which point the moment is, $M = 2245 \times 54.62 - 748 \times 24 - 374 \times 48 = 86100$ inch lb.

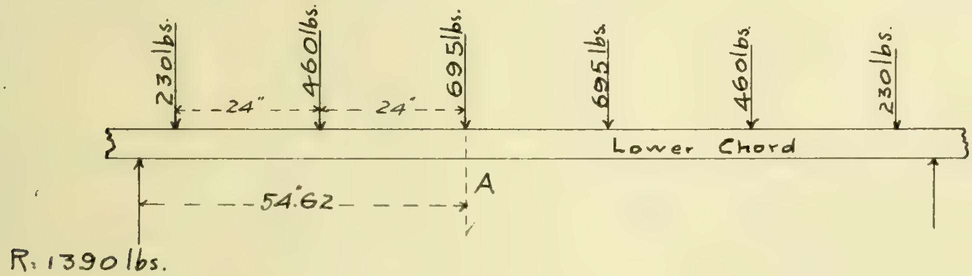
$S = \frac{86100 \times 2\frac{1}{8}}{6.4} = 28600$ lb. per sq. in. On net section = 33500 lbs per sq. in.

Direct dead load stress = 7300 " "

Crossbending dead load stress $\underline{= 5220}$ " "

Total stress = 46020 " "

When crossing a bridge a traction engine would have, ordinarily, the clear right of way and would keep in the middle of the bridge. In this case the maximum floor beam reaction is $\frac{2}{3} \times 8000 \times \frac{1}{2} = 278$ lbs. The load is distributed as before, but the amount of the loading on one truss being reduced, the stresses will also decrease.



Moment at A:

$$M = 1390 \times 54.62 - 460 \times 24 - 230 \times 48$$

$$= 53600 \text{ inch lbs.}$$

$$S = \frac{53600 \times 2\frac{1}{8}}{6.4} = 17800 \text{ lbs. per sq. in.}$$

On net section this stress = 17800 × 1.17 = 20800 lbs. per sq. in.

Direct dead load = 7300 " "

Cross bending dead load stress = 5220 " "

Total stress = 33320 " "

Direct live load = 1100 " "

Total stress = 34420 " "

$\frac{14350}{34420} = .41$ Efficiency.

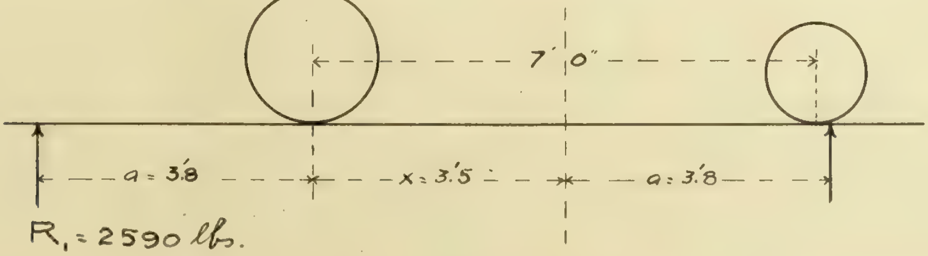
Case 3.

An ordinary wagon will have a gage of about 5 feet, and an average distance between points of contact of the wheels of about 7 feet. The maximum load will be, the weight of the wagon, of the driver and of the load; since the panel length is but 11 feet the horses will be in the next panel.

Load of 60 bushels of corn @ 70 lbs.	= 4200 lb.
Wagon and driver	= 1600 "
Total	= 6800

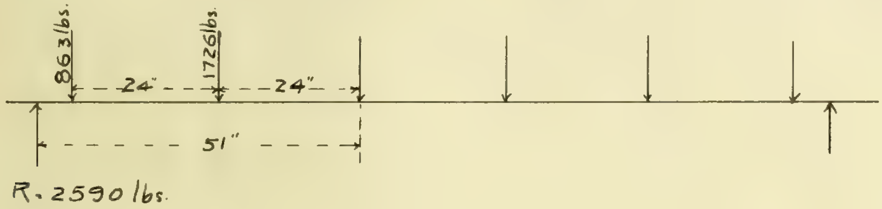
Call the maximum load 6000 lbs. equally transferred by each of the four wheels. In order to have the maximum loading, two maximum loads must be on the panel at the same time. Suppose one load is one of corn going to town and the other a load of coal going out of town. There will then be a load of about 12,000 lbs. on the panel. The load will be equally divided between the two trusses, each receiving 6000 lbs.

Position for maximum moment:



On the diagram of the floor system, if the wagon wheels are at W, the reaction at Q is 2590 lbs., and the maximum moment is under floor beam J_3.

$M = 2590 \times 51 - 1726 \times 24 - 863 \times 48 = 47400$ inch lbs.



$S = \dfrac{47400 \times 2\frac{1}{8}}{6.4} = 15750$ lbs. per sq. in.

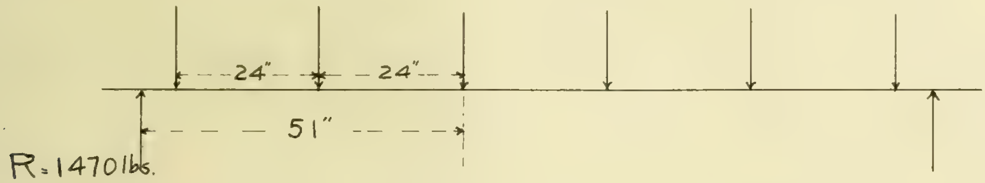
On net section $S = 15750 \times 1.17 = 18200$ lbs. per sq. in.

Direct dead load stress	= 7300	" " "
Crossbending " " "	= 5220	" " "
Direct live " "	= 3180	" " "
Total stress	= 33900	" " "

$\dfrac{14350}{33900} = .42$ Efficiency.

Case 4.

When but one wagon is considered, its center of gravity will travel along a line about 4 feet from the center line of the nearest truss. The load taken by this truss will be $\frac{14-4}{14} \times 6000 = 4300$ lbs.



$$R = \frac{2150 \times 7.3 + 2150 \times .3}{11.1} = 1470 \text{ lbs.}$$

$$M = 1470 \times 51 - 980 \times 24 - 490 \times 48 = 28000 \text{ inch lbs.}$$

$$S = \frac{28000 \times 2\frac{1}{8}}{6.4} = 9300 \text{ lbs. per sq. in.}$$

On net section, $S = 9300 \times 1.17 = 10900$ lbs. per. sq. in.

Direct dead loadstress	= 7300	" " "
Crossbending " " "	= 5220	" " " "
Direct live " "	= 2300	" " " "
Total stress	= 25720	" " " "

$\frac{14350}{25720} = .55$ Efficiency

The stresses produced by applying the assumed live load, have shown the loading to be excessive as regards the lower chord, but for the other members it is safe. The middle panel of the lower chord is stressed to 12500 pounds per square inch by the dead load alone, leaving but a narrow margin for live load. The arched upper chord is safe for the assumed live load and for any load likely to come upon the bridge, the middle panel only showing any weakness, which is due mostly to its eccentricity. The stress produced in this portion of the arch by a 15800-pound traction engine is far below the allowable stress.

The vertical and diagonal members are all capable of carrying the loads imposed upon them, the maximum unit stresses produced in them being below the allowable stress by a large margin.

The really weak portion of the bridge is the lower chord. This member is entirely out of proportion to the others.

The method employed in finding the stresses in the several members of the truss takes no account of any action except of that as a chord, that is to say, the arch action and the beam action are not accounted, the arch sections being considered as pin-connected. The beam action can not be great, since it is unreasonable to expect a beam one hundred feet long and only ten inches deep to carry any considerable load. The arch action is probably small, but it may assist some by increasing the rigidity of the bridge and by relieving, to a slight extent, the stresses in the lower chord.

In treating the sections of the arch, each   is considered as a column with fixed ends, which is reasonable since the arch is continuous.

The arrangement of the floor system has a weakening effect on the lower chord. This arrangement is shown in the diagram page 41. Since the floor beams rest directly upon the lower chord, the load is applied as concentrations every two

feet, producing large cross-bending stresses. In the case of the dead load alone the cross-bending stress is 5220 pounds per square inch, or over two-thirds of the direct dead load stress.

The stresses produced in the lower chord by a 15800-pound traction engine are above the ultimate strength of the material. The whole floor system is too weak for such a loading as this, the stress produced in the floor beams is in excess of 3100 pounds per square inch.

In applying loads on the lower chord only the middle panel is used since it is 11.1 feet long exceeding the length of any other panel by about one foot. If none but direct stresses were considered in regard to the lower chord, they would not exceed reasonable limits. It is the cross bending due to arrangement of floor system which reduces the efficiency of, and produces such large stresses in the lower chord.

The bridge is located on one of the principical roads leading into Mahomet, hence the traffic which it carries is a little more than the ordinary. It is said that the bridge has not carried traction engines for about ten years.

It is doubtful whether the stress in the lower chord often exceeds 25000 pounds per square inch. In case an ordinary traction engine should be taken across, the bridge would probably carry it, but at the same time the metal in the lower chord would be severely strained and perhaps permanently injured. As shown in the computations, the 8000-pound engine produces a stress of over 35000 pounds per square inch. This stress is above the elastic limit of the material. The effect on the metal would be to raise its elastic limit and its ultimate strength, but it would reduce the number of times it would be capable of sustaining such a stress. This is explained in the extract from an article by Prof. J. B. Johnson, entitled, "Testing the Strength of

Engineering Materials", given on page .
The metal would not often be subjected to such a stress as the above, and in the intervals between such loads it would have time to recover, as explained in Prof. Johnson's paper.

The number of repetitions of a stress of 35000 pounds per square inch which the metal could sustain is shown in the results of Wöhler's experiments part of which are given in tables 3 and 4 on page 39.

These experiments show that for a stress of 35,000 pounds per square inch the metal would withstand at least 38000 repetitions, and that it would withstand a repeated stress of 31,600 pounds per square inch indefinitely. There would be comparatively few such stresses as these and they would be well scattered. It is not reasonable to expect the metal, which has been exposed for many years to the weather and other weakening influences to work so far above its elastic limit without rupturing, when subjected to

a far less number of repetitions of such stresses as those above. The maximum load which the metal would carry can not be determined. It is possible that it could be strained to 40,000 pounds per square inch and yet stand. Such a proceeding, however, would no doubt be dangerous and exceedingly severe upon the material.

It is also shown, in the computations, what the effect would be when two maximum wagon loads (6000 pounds each) are considered as passing on the middle panel of the bridge. Of course this is an extreme case and quite unlikely to occur. Here again the stress goes above the elastic limit of the material, and the same conditions hold as for the 8000-pound traction engine.

In the case of a single 6000-pound wagon-load the stress reaches over 25000 pounds per square inch. Although this stress is above a reasonable working limit the metal would carry it. Even this load causes a stress in excess of what ordinarily comes upon the material. Very few wagons with

their loads, as much as 6000 pounds, and the majority of the vehicles which cross the bridge are far lighter.

Nothing has as yet been said as to the effects of impact and shaking by loads and by the wind. When an empty wagon is drawn across the bridge at a walk, the shaking and jolting produced fills a person, who is standing on the bridge, with an uncomfortable feeling of insecurity. There seems to be no rigidity to the structure. The movements under a load are not short, quick, and sharp as in a stiff new bridge, but of a long swaying nature. This action must have some effect upon the riveting and upon the main members, and it can not be a good effect. It is to be remembered too, that this jolting is transferred to that weak member, the lower chord, by the floor system as bending stress, which is undesirable.

The material in the lower chord has been subjected to stresses considerably in excess of what is allowable and safe. Al-

though the experiments referred to show what the metal has stood in tests, they do apply so readily to metal which has been exposed, unpainted, to the weather and to large working stresses. The strength of the metal and the abuse it will resist, as shown by the experiments, account readily enough for why the bridge stands. The present condition of the metal is unknown, but it is reasonable to expect that it has deteriorated and has become incapable of resisting, with safety, the strains which are likely to come upon it.

Appendix.

Testing the Strength of Engineering Materials

By Prof. J. B. Johnson, in Jour. Assoc. Eng. Soc. Vol. 7 pp. 92.

* * "Elaborate experiments made in Germany, and elsewhere, have shown that the ultimate strength of a metal from a single test is no indication of its ability to resist repeated stresses".

* * * * * * *

"The opinion is currently held, even among engineers, that if a member of a structure, or a piece of metal, has been strained or distorted beyond its elastic limit, that it has been permanently injured. There is no truth whatever, in this belief, further than that the permanent distortion may unfit the member for the purpose for which it was originally intended. The metal itself has not only not been injured, but for many purposes it has been improved. It is not generally known that <u>any metal is (almost) perfectly elastic up to the limit of its greatest previous loading</u>. The word almost is put in parenthesis to indicate that it is so very nearly

perfectly elastic that for one or two or three repetitions no appreciable lengthening would be observable from repeating the maximum load, whereas if many thousands of repetitions should be made the member would be found to lengthen slightly. Thus if a piece of wrought iron whose elastic limit is found at 30,000 pounds to the square inch, be loaded to 40,000 pounds to the square inch, the elastic limit has now become 40,000, or if loaded to 45,000, it is found to be (nearly) perfectly elastic up to 45,000 pounds. Thus if it is desired to use a given iron rod in a machine or structure, for a limited number of repetitions, say, less than 10,000, up to a unit stress of, say, 40,000 pounds to the square inch, without its being distorted in its use, it would only be necessary to load it, in a testing machine, or otherwise to, say, 45,000 pounds to the square inch, when it would be found as elastic as steel up to the working limit of 40,000 pounds. This operation is called <u>taking the stretch</u> out of the metal, just as we speak of taking the stretch out of a rope. If this metal were

now to be subjected to a stress of 30,000 pounds per square inch, it would not carry this stress as many times as if it had not been already loaded too high. That is to say, there is less capacity for work left in the piece than if the stretch had been left in it. It is only in machines that many hundreds of thousands of repetitions of maximum stresses occur, and hence it is here that the fatigue of metals is of most importance. In a roof truss, for example, where the maximum loads are never repeated perhaps as many times as the structure will see years of service, it would be a matter of no consequence if the iron members had at one time been loaded far beyond their elastic limits. The same would hold for highway bridges and perhaps for railway bridges, since the <u>maximum</u> loads seldom come on such structures.

"<u>Another effect</u> of loading a member beyond its elastic limit is that <u>it greatly increases its ultimate strength and diminishes its total elongation</u>. That is to say, if if 50,000-pound iron be stretched to 40,000

or 45,000 pounds and there left for several days or weeks, and again tested, it will show a strength of perhaps 60,000 pounds, but the total elongation will be perhaps only 15 or 18 per cent, instead of 25 per cent its normal elongation in a single test.

"Since presenting the preceding the writer has made the following tests on $1\frac{1}{8}$ inch square bar iron. The bars were put in a machine without preparation and broken. After periods of 1, 7, and 22 days, ends of the broken specimens, which had been strained up to the breaking limits on the first tests, were again put in and broken with the following results:

Effect of Straining Iron Beyond its Elastic Limit.

No. of Specimen	Date of First Test	Date of Second Test	Unit Strength First Test	Unit Strength Second Test	Period of Rest Days	Increase in Strength	Per Cent of Increase of Strength
VI	Jan. 26	Jan. 27	51,040	58,300	1	7,260	16.9
IX	"	"	49,150	58,800	1	8,540	16.9
X	"	"	50,400	58,940	1	9,650	16.9
I	"	Feb. 2	49,940	61,750	7	11,810	22.6
XV	"	"	50,550	61,450	7	10,900	22.6
XVII	"	Feb. 17	49,150	62,060	22	12,910	26.8
XXII	"	"	48,270	61,500	22	13,230	26.8

"The total reduction in area was considerably less on the second break than it was on the first. The average elongation on the first tests was 22 per cent in eight diameters and the average reduction of area was 35 per cent. The average elastic limit was 28,000 pounds per square inch."

Table 3.

Wohler's Experiments on Bars Subjected to Repeated Tensions between Definite Limits.*

Num. of Bar	Form of Bar†	Material	Stress applied in lbs. per sq. in. Maximum	Minimum	Range of Stress in lbs. per sq. in.	No. of Repetitions before Fracture
1	A	Iron Axle, Phoenix Company.	45,840	0	45,840	800
2	A		42,020	0	42,020	106,910
3	A		38,200	0	38,200	340,853
4	A		34,380	0	34,380	409,481
5	A		34,380	0	34,380	480,852
6	A		30,560	0	30,560	10,141,645
7	A		+42,020	+19,100	22,920	2,273,424
8	A		+42,020	+22,920	19,100	Not broken [4,000,000]
9	B		34,200	0	34,200	37,828

*Unwin's Materials of Construction pp. 365.

†The bars marked A had well-rounded corners at the point where the small middle joined the enlarged end. Those marked B had square corners. It may be noted at once that for any given stress the bars B broke with far fewer repetitions of stress than the bars A.

Table 4.

Limits of Stress for Unlimited Repetition of Load (Wöhler)*

Bars subjected to simple tension, compression, or bending

Material	Stress in lbs. per sq. in. Maximum	Minimum	Range of Stress
Wrought Iron	+15,300	− 15,300	30,600
do.	+ 31,600	0	31,600
do.	+ 42,000	+ 23,000	19,000

*Unwin's Materials of Construction pp. 372

The arrangement of the floor system is shown in the diagram on page . The flooring is two inches thick and it rests on 3"×4" stringers, of which there are fourteen to the width of the bridge. The stringers are supported by 2"×12" floor beams spaced 2'-0" apart, which rest directly upon the lower chord. This arrangement, while poor, has a redeeming feature in that it distributes the load. As shown in the computations, the flooring is capable of transferring the assumed loads to several stringers without exceeding a reasonable unit stress, and in turn the stringers perform a similar office by carrying the loads safely to the floor beams. By the time the load reaches the lower chord, it is well distributed along the panel. Without this distributing action of the floor system, the stresses in the lower chord would be nearly doubled.

Diagram of Floor System.